SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Revised Geotechnical Report
King Street Station
Fourth Avenue Retaining Wall
Seattle, Washington

June 29, 2006

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

At Shannon & Wilson, our mission is to be a progressive, well-managed professional consulting firm in the fields of engineering and applied earth sciences. Our goal is to perform our services with the highest degree of professionalism with due consideration to the best interests of the public, our clients, and our employees.

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21-1-20406-002

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REVISED GEOTECHNICAL REPORT
KING STREET STATION
FOURTH AVENUE RETAINING WALL
SEATTLE, WASHINGTON

1.0 INTRODUCTION

This report presents the results of our field explorations and geotechnical engineering studies for the construction of a retaining wall at the Interstate 90 (I-90) on-ramp at Fourth Avenue South in Seattle, Washington. The purpose of our work was to characterize the backfill conditions of the existing wall and to use other existing subsurface data to provide geotechnical recommendations for design of a replacement wall.

The work was completed in general accordance with our agreement with HDR dated September 16, 2005 (WSDOT On Call Rail Engineering and Operations Services Agreement Y-9383 Subconsultant Agreement); Addendum #1 dated October 5, 2005; and a proposal for two borings at the Fourth Avenue on-ramp dated October 18, 2005.

2.0 SITE AND PROJECT DESCRIPTION

The project is located on the west side of the I-90 eastbound on-ramp from Fourth Avenue South (Figure 1). The area is relatively flat, with existing ground surface elevations of about 18 to 20 feet, as shown in Figure 2. An existing mechanically stabilized earth (MSE) retaining wall with concrete facing supports the west side on-ramp. The purpose of this project is to replace the existing retaining wall with another wall located approximately 20 feet to the east from Station 6+60 to 10+ 06 to make room for an additional railroad track. New sign foundations and a moment slab will also be constructed.

3.0 FIELD EXPLORATIONS AND LABORATORY TESTING

Two geotechnical borings were drilled on November 11, 2005, at the locations shown in Figures 2 and 3 to characterize the subsurface conditions behind the existing retaining wall. Borings BH-1 and BH-2 were drilled to depths of approximately 24 feet each. Holt Drilling performed the drilling under subcontract to Shannon & Wilson, Inc., using mud-rotary techniques. A representative of Shannon & Wilson, Inc., observed the drilling and sampling of the borings.

In conjunction with obtaining soil samples, Standard Penetration Tests (SPTs) were performed at regular intervals, as shown in the boring logs in Figures 5 and 6. A soil classification and log key is presented in Figure 4. The SPTs were performed in general accordance with the American Society for Testing and Materials (ASTM) D 1586, Test Method for Penetration Test and Split-Barrel Sampling of Soils. In the SPT, a 2-inch outside-diameter (O.D.), 1.375-inch inside-diameter (I.D.), split-barrel sampler is driven with a 140-pound hammer falling through a height of 30 inches. The number of blows required to achieve each of three 6-inch increments of sampler penetration is recorded during the test. The number of blows for the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). When penetration resistances exceed 50 blows for 6 inches or less of penetration, the test is terminated and the number of blows is recorded along with the penetration. The N-values for the tests performed in the borings are presented graphically on the boring logs in Figures 5 and 6.

Soil samples retrieved from the split-barrel sampler were logged, visually classified, sealed in jars, and returned to our laboratory in Seattle for further classification and testing.

Laboratory tests were performed on selected soil samples retrieved from the borings. The laboratory testing program included visual classification and tests to determine the natural water content. Classification of the samples was generally based on ASTM D 2487, Standard Test Method for Classification of Soil for Engineering Purposes, and ASTM D 2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure).

Water content was determined in general accordance with ASTM D 2216, Test Method for Determination of Water (Moisture) Content of Soil and Rock. The water content is shown graphically on the boring logs (Figures 5 and 6).

4.0 SUBSURFACE CONDITIONS

A generalized subsurface profile was developed based on the field explorations, as shown in Figure 3. Below approximately 2½ inches of Portland cement concrete pavement, we encountered 18 to 22 feet of dense to very dense, slightly silty, gravelly sand embankment fill with a trace of cobbles. The fill thickness increases with increasing height of the existing wall. We note that the very high N-values recorded in this fill layer are likely the result of gravel and cobbles impeding the penetration of the SPT sampler, rather than a true indication of the density of the fill.

In boring BH-2, the boring penetrated below the sand fill behind the retaining wall into soft, silty clay. Based on our experience in the project area and explorations by others, the soft clay and soft/loose silt below the wall are likely to extend to depths of about 60 to 70 feet below the ground surface. The soft soils are underlain by glacial till.

We did not encounter groundwater in our explorations. Based on our project experience in the area, the groundwater depth below the existing ground surface is likely to range from about 4 to 6 feet. The actual depth to groundwater would vary with seasonal fluctuations in precipitation, tidal influences, and other factors.

5.0 ENGINEERING STUDIES AND RECOMMENDATIONS

5.1 General

Based on the results of our subsurface explorations behind the existing retaining wall and our laboratory tests, we performed geotechnical engineering studies to develop recommendations for the design and construction of a replacement soil nail wall.

5.2 Seismic Design Recommendations

5.2.1 Ground Motions

We understand that the wall will be designed in accordance with the 2004 American Association of State Highway and Transportation Officials (AASHTO) Load Resistance Factor Design (LRFD) Bridge Design Specifications and the Washington State Department of Transportation (WSDOT) Geotechnical Design Manual (M46-03). AASHTO and WSDOT criteria indicate that bridge design and evaluations should be based on earthquake ground motions with a 10 percent chance of exceedance in 50 years (475-year return period) for non-critical transportation structures.

The U.S. Geological Survey (USGS) completed regional probabilistic ground motion studies and published ground motion maps for the entire country in 2002. These USGS maps, with modifications, have subsequently been incorporated into the WSDOT Geotechnical Design Manual and Bridge Design Manual. For a recurrence interval of 475 years, the site peak ground acceleration (PGA) on rock is approximately 0.33g at the location of the wall. Consequently, we recommend that a site PGA for rock of 0.33g be used in the seismic design of the wall.

Based on existing information regarding the soils at the site, we recommend that the site be classified as AASHTO Soil Profile Type IV with a corresponding site factor of 2.0. AASHTO describes a Soil Profile Type IV as a deposit of soft clay or silt greater than 40 feet in thickness.

5.2.2 Earthquake Hazards

Earthquake-induced geologic hazards that may affect a given site include fault rupture, landsliding, and liquefaction and associated effects (settlement, loss of shear strength, bearing capacity failures, loss of lateral support, ground oscillation, lateral spreading, etc.). The potential for occurrence of these various hazards was evaluated as described in the following paragraphs.

5.2.2.1 Fault Rupture

The project area lies near the Seattle Fault Zone with the northernmost strand of the fault zone/deformation front mapped 1,000 feet to the south of the site. However, the recurrence interval for large earthquakes capable of rupturing the ground surface in this zone appears to be on the order of thousands of years (e.g., Nelson et al., 2003a, 2003b), much longer than the 475-year return period specified for seismic design by AASHTO. Therefore, the relative risk posed by ground surface fault rupture at the site is low.

5.2.2.2 Landsliding

The gentle slopes in the vicinity of the site result in a low landsliding potential.

5.2.2.3 Liquefaction and Associated Effects

The project area is mapped as having high liquefaction potential; however, the liquefaction potential was evaluated by Hart-Crowser on a site-specific basis in 1986. Since 1986, the procedures for liquefaction evaluation have been slightly updated. Re-evaluation of the liquefaction hazard was not included in our scope of work.

5.3 Soil Nail Wall

5.3.1 General

A permanent soil nail retaining wall is proposed between Stations 6+60 and 10+06. In general, the wall cut will be behind the existing MSE wall in dense to very dense embankment fill overlying soft and loose fill and alluvial soils. The embankment fill consists of gravelly sand with occasional cobbles. As currently designed, the soil nails will be permanent with a temporary shotcrete facing. A cast-in-place concrete facing will be formed and poured or shotcreted onto the front of the temporary shotcrete facing.

During subsurface explorations, groundwater was not encountered within the depth of the proposed soil nail wall excavation. The embankment fill material was moist and the natural water content tests indicate the fill is near the optimum moisture content.

5.3.2 Description of Soil Nailing

Soil nailing consists of drilling and grouting a series of steel bars or "nails" behind the excavation face and then covering the face with reinforced shotcrete. The placement of relatively closely spaced steel nails in the retained soil mass increases the shear resistance of the soil against rotational sliding, increases the tensile strength of the soil behind potential slip surfaces, and moderately increases shear resistance at a potential internal slip surface due to the bending stiffness of the nails.

Soil nailing is most effective in dense, granular soils and stiff, massive, low plasticity, fine-grained soils that exhibit adequate "stand-up" time prior to placing shotcrete. Soil nailing may not be cost-effective in loose granular soils, soft cohesive soils, highly plastic clays, or where uncontrolled groundwater exists above the bottom of the excavation. In general, up to 8-foot vertical excavation faces must be able to stand unsupported for 24 hours in order for soil nailing to be feasible. The length of exposed cut faces will depend on actual encountered soil and groundwater conditions.

Soil nails consist of steel bars (typically ¾- to 1⅜-inch-diameter), which are installed by tremie grouting the nail into a predrilled hole. Casing and a down-the-hole hammer may also be required locally to install soil nails through dense cobbles. Soil nails are located in a rectangular or triangular grid pattern and are typically installed at a declination angle of 15 degrees from horizontal. The construction sequence of a soil nail wall generally includes three steps:

(1) staged excavation, (2) nail installation and select nail testing, and (3) drainage and facing construction. This sequence is repeated until the excavation and shoring are complete.

5.3.3 Anticipated Movements

Soil nailing is a passive shoring system and develops capacity when the retaining wall deflects toward the excavation and the nails are mobilized in tension. Excessive deflection could result in damage to structures and utilities adjacent to the excavation. Our experience has shown that lateral deflections with soil nail walls of similar and greater height, and in similar soils as those anticipated at the site, are typically ¾ to 1 inch. Similar vertical settlements are expected to occur at the face of the wall. Vertical settlements will decrease with distance from the wall and should be negligible beyond a distance of about the wall height. Wall monitoring during construction is recommended (Section 6.3).

5.3.4 Soil Nail Wall Design Parameters

The recommended lateral pressures for design of the permanent soil nail wall are presented in Figure 7. Additional surcharge loading pressures are presented in Figure 8. Surcharge used in soil nail wall design was a uniform 400 pounds per square foot (psf) for traffic and a uniform 900 psf for barrier block and sign near Station 8+40.

The static and dynamic earth pressures presented in Figure 7 are based on a friction angle of 37 degrees and a unit weight of 130 pounds per cubic foot (pcf). The presence of gravel and cobbles in the embankment fill made it difficult to estimate the friction angle of the fill from measured SPT N-values. Thus, we estimated a friction angle of 37 degrees for the embankment fill based on visual observation of the soil retrieved during the field explorations and our local experience with compacted fills.

The dynamic earth pressure increment presented in Figure 7 is based on the recommendations presented in the WSDOT Geotechnical Design Manual (GDM). The PGA on soil at the site was estimated from the deamplification factor for Site Class E in Table 6-3 of the GDM. This resulted in an estimated PGA on soil of 0.32g and a corresponding horizontal acceleration coefficient (k_h) of 0.16.

The Mononobe-Okabe method and Rankine earth pressure theory were used to estimate the dynamic earth pressure increment (ΔKae). The seismic increment was calculated as Kae - Ka. The dynamic increment is shown in Figure 7 as an inverted trapezoid with the pressure at the top of the distribution equal to 0.8 $\Delta Kae\gamma$ (wall height) and the pressure at the bottom equal to 0.2 $\Delta Kae\gamma$ (wall height).

Parameters used in the design of the soil nail wall were as follows:

- Soil Unit: Dense Fill
- Moist Unit Weight: 130 pcf
- Angle of Internal Friction: 37 degrees
- Soil Cohesion: 50 psf
- Ultimate Soil Nail Pullout Resistance: 6.0 kips per lineal foot

Based on our experience in similar dense granular soils near optimum moisture content, the soil would exhibit some cohesion during the cut face excavation.

Soil nail wall design stability calculations were performed in accordance with the 2003 Federal Highway Administration (FHWA) Manual for Design and Construction of Soil Nails using the computer program GoldNail v.3.11 (1996).

In accordance with the referenced FHWA manual, the following partial factors of safety were used in the analysis of internal and external wall stability:

Design Component	Partial Factor-of-Safety	
	Static	Seismic
Soil Friction	1.50	1.10
Soil Cohesion	1.50	1.10
Soil-Grout Adhesion	2.00	1.50
Sliding	1.50	1.10
Bearing	3.00	2.30
Nail Bar Yield Strength	1.80	1.35
Facility Capacity	1.50	1.10

For the interim construction conditions where excavation for a lift has occurred and the corresponding nail row has not yet been installed, the required partial factors of safety for soil friction and cohesion were reduced to 1.20 in accordance with the referenced FHWA manual.

5.4 Bearing Resistance

We understand that bearing pressures are necessary to evaluate the proposed moment slab located immediately above the soil nail wall. The moment slab will be 7 feet wide from the wall face and have a thickness of 1 foot 7 inches. The base of the slab will be 2½ feet below the proposed pavement surface at the top of the wall. As mentioned previously, the soil conditions below the ground surface surface (top of wall) are dense to very dense embankment fill overlying soft and loose fill and alluvial soils. The thickness of the embankment fill overlying the soft and loose soils increases along the alignment as the stationing and ground surface elevation increase.

We understand that the moment slab will be designed using load resistance factor design (LRFD) design methodology. Bearing resistance for LRFD design is determined by applying the appropriate resistance factor (Rf) to the nominal bearing resistance (for the appropriate limit state). Recommended Rf values for the various limit states (for shallow foundation design) are presented in Table 1, Recommended Resistance Factors for Spread Footing Design.

Based on the available subsurface information, it appears that the soft and loose soils underlying the site are within the depth zone considered to provide bearing resistance for the proposed slab. The contribution of bearing resistance provided by the soft soil layer increases, as the thickness of the embankment fill below the ground surface decreases. Accordingly, and in accordance with the WSDOT GDM, we have estimated the nominal bearing resistances at four different locations along the alignment. Our recommended values for design are presented in Table 2, Recommended Nominal Bearing Resistance Values for Spread Footing Design, and in Figure 9, Nominal Bearing Resistance of Moment Slab versus Station Location.

5.5 Lateral Resistance

Lateral forces would be resisted by passive earth pressures acting against buried portions of a structure, and friction along the bottom of the structure. In our opinion, passive earth pressures developed from the dense to very dense embankment fill could be based on an equivalent fluid weight of 1,150 pcf. This value is based on the assumptions that the structure extends at least 2 feet below the lowest adjacent exterior grade, proper drainage is implemented, and that the backfill around the structure is compacted in accordance with the WSDOT specifications. We recommend that a resistance factor of 0.5 be applied to the above equivalent fluid unit weight for design using the LRFD method.

To evaluate friction along the bottom of footings or a slab, we recommend that a coefficient of friction of 0.45 be used between cast-in-place concrete and soil. We recommend that a resistance factor of 0.8 be applied to this coefficient for design using the LRFD method.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Construction Monitoring

Geotechnical design recommendations are developed from a limited number of explorations and tests. Therefore, recommendations may need to be adjusted in the field. We recommend that Shannon & Wilson, Inc., be retained to monitor the geotechnical aspects of construction, particularly installation of the soil nails. This monitoring would allow us to determine that the work is accomplished in accordance with our recommendations.

6.2 Soil Nail Wall

Soil nails should be installed in a horizontal sequence with the base of the staged excavation extending a maximum of 2 feet below the level of the nail to be installed. Any utilities to be removed or installed behind the soil nail retaining wall should be completed prior to wall excavation.

Based on our experience, we anticipate that little sloughing will occur in the compacted embankment fill if the soils are dry and unsupported heights do not exceed 8 feet. However, if the soil does not contain sufficient binder or fine material, it may slough; no test cuts were completed during our study. To minimize ground loss at the excavation face, it may be necessary to leave a shallow stabilizing berm in front of the wall. Soil nails would then be installed through the stabilizing berm. The berm must lie below the previous shotcrete lift and be constructed at a safe slope. After nails are installed through the berm, the berm would be excavated around the nails, and reinforced shotcrete would be applied.

The Contractor should anticipate gravel and cobbles at the wall location and should be prepared to remove any cobbles or other obstructions during mass excavation that protrude into the retaining wall face of the excavation. The voids produced by removing face obstructions should be backfilled with shotcrete or controlled density fill (CDF).

6.3 Soil Nail Wall Monitoring

Survey monitoring of the soil nail retaining wall should be performed. Surveys should be made to determine the plan location and elevation of wall monitoring points before the start of construction and on a regular basis throughout construction to evaluate movements resulting from construction activity.

In general, wall monitoring points should be surveyed and the results reported to WSDOT twice weekly during construction and once monthly after construction. Unless excavation progress or post-construction results dictate otherwise, less or more frequent surveys could be determined by the engineer. If any movements exceed ½-inch, monitoring points should be surveyed daily in the areas of concern. If any movements exceed ½ inch between successive surveys during construction, excavation progress in the areas of concern should be halted and remedial measures implemented. However, if wall movements appear to have stopped increasing at any monitoring point, then monitoring frequency may be reduced at the discretion of the engineer.

Monitoring of the soil nail wall should consist of controlled surveying of the elevation and plan location of survey points placed at the top of the soil nail walls (attached to the CIP fascia wall) and spaced no more than 20 feet on center along the length of the wall.

Monitoring points should consist of bolts or rods embedded into the object of interest, or cross-hairs scribed onto a plate that is attached to the face of the object of interest. Accuracy of the survey monitoring shall be ±0.005 foot.

6.4 Wet Weather Considerations

In the Seattle area, wet weather generally begins about mid-October and continues through about May, although rainy periods could occur at any time of year. Therefore, it would be advisable to schedule earthwork during the dry weather months of June through September. The soils encountered during explorations that are likely to be encountered during grading activities are granular but contain sufficient amounts of silt and fine sand to make them moisture-sensitive. The soils would likely provide a suitable working surface under dry conditions; however, after continual repetitions by wheel loads, the materials could degrade, especially in the presence of water.

In addition, during wet weather months, the groundwater levels could increase, resulting in seepage into excavations. Performing earthwork during dry weather would reduce these problems and costs associated with rainwater, trafficability, and handling of wet soil. However, should wet weather/wet condition earthwork be unavoidable, the following recommendations are provided:

- The ground surface in and surrounding the construction area should be sloped as much as possible to promote runoff of precipitation away from work areas and to prevent ponding of water.

- Work areas or slopes should be covered with plastic. The use of sloping, ditching, sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soils and placement and compaction of clean structural fill could be accomplished on the same day. The size of construction equipment could need to be limited to prevent soil disturbance. It could be necessary to excavate soils with a backhoe, or equivalent, and locate them so that equipment does not pass over the excavated area. Thus, subgrade disturbance caused by equipment traffic would be minimized.
- Fill material should consist of clean, well-graded, pit-run sand and gravel soils, of which not more than 5 percent fines by dry weight passes the No. 200 mesh sieve, based on wet-sieving the fraction passing the ¾-inch mesh sieve. The gravel content should range between 20 and 50 percent retained on a No. 4 mesh sieve. The fines should be nonplastic. Well-graded recycled concrete is a suitable alternative.
- No soil should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible.
- In-place soil or fill soil that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced with clean, granular soil (see gradation requirements above).
- Excavation and placement of structural fill material should be observed on a full-time basis by a representative of our firm experienced in wet weather/wet condition earthwork to determine that all work is being accomplished in accordance with the project specifications and our recommendations.
- Grading and earthwork should not be performed during periods of heavy, continuous rainfall.

We recommend that the above requirements for wet weather/wet condition earthwork be incorporated into the contract specifications.

7.0 LIMITATIONS

The analyses, conclusions, and recommendations presented in this report are based on interpreted site conditions. We further assume that the exploratory test borings are representative of the subsurface conditions throughout the site; i.e., the subsurface conditions are not significantly different from those encountered in the explorations. If subsurface conditions different from those encountered in the explorations are observed or appear to be present during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between submission of our report and the start of work at the site, or if conditions have changed because of natural forces or human activity, or if conditions appear to be different from those described in our report, we recommend that we review this report to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

Within the limitations of scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practices in this area at the time this report was prepared. We make no other warranty, either express or implied. These conclusions and recommendations are based on our understanding of the project as described in this report and the site conditions as interpreted from the previous exploratory test borings.

This report was prepared for the exclusive use of HDR, WSDOT, and the design team of the Fourth Avenue I-90 on-ramp retaining wall. It represents factual data, our engineering recommendations, and construction considerations based on experience and is not a warranty of subsurface conditions, such as those interpreted from the boring logs and discussions of subsurface conditions included in this report.

Unanticipated conditions are commonly encountered and cannot be fully determined by merely taking soil samples or making explorations. Such unexpected conditions frequently require additional services to achieve a properly constructed project. Some contingency fund is recommended to accommodate such potential extra costs.

The scope of our services included no environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous/toxic substances in the soil, surface water, groundwater, or air, on or below or around the site, or for the evaluation/disposal of contaminated soils or groundwater should any be encountered during construction.

Shannon & Wilson has prepared and included an Appendix, "Important Information About Your Geotechnical Report," to assist you and others in understanding the use and limitations of our reports.

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TABLE 1
RECOMMENDED RESISTANCE FACTORS FOR SPREAD FOOTING DESIGN

Limit State	Resistance Factor			
	Bearing	Shear Resistance to Sliding		Passive Pressure Resistance to Sliding
Strength	0.45	0.9 (Pre-cast)	0.8 (Cast-in-place)	0.50
Service	1.0	-	-	-
Extreme Event	1.0			

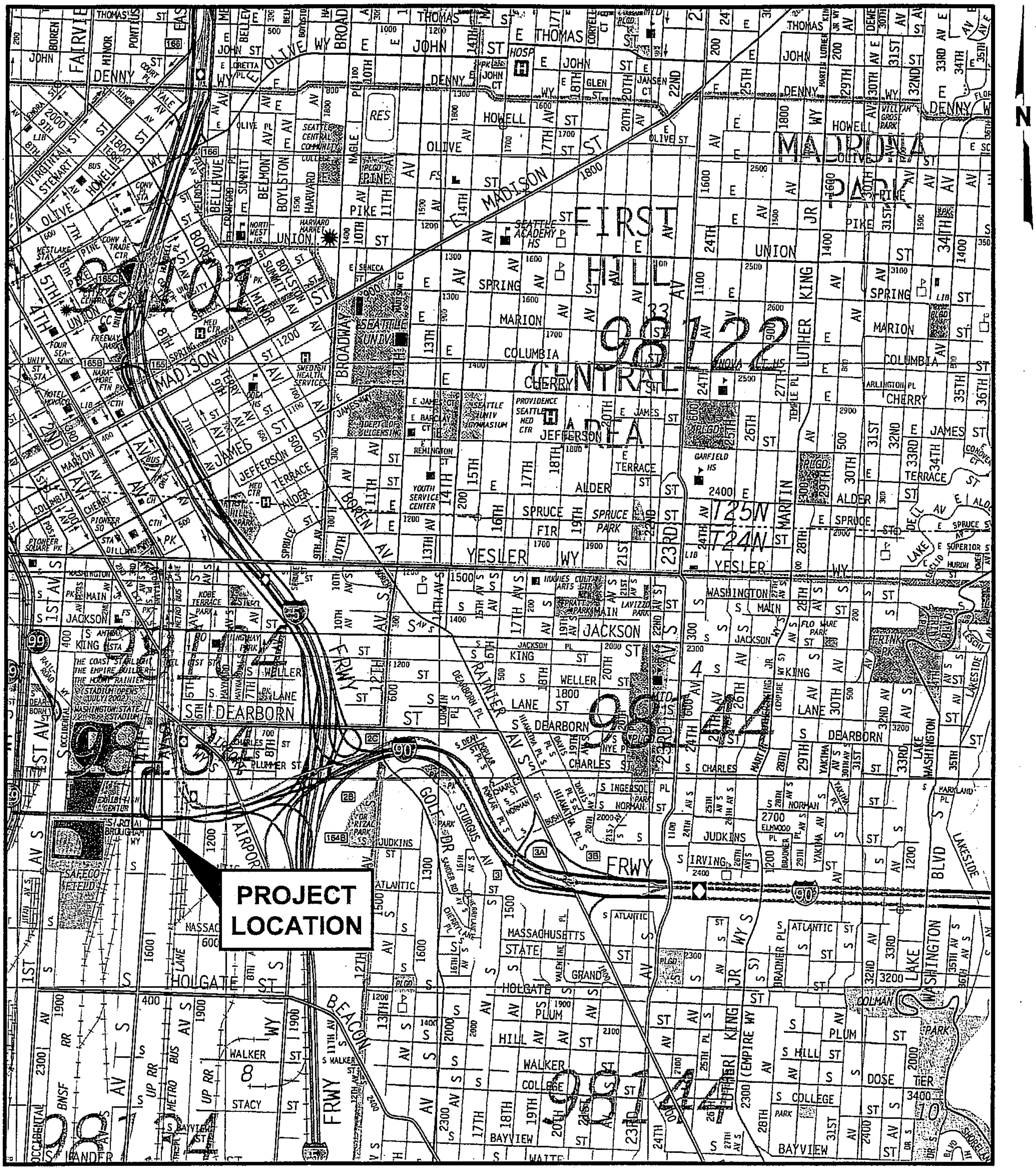
Note:
We recommend that a coefficient of friction of 0.45 be used between cast-in-place concrete and soil.

TABLE 2
RECOMMENDED NOMINAL BEARING RESISTANCE VALUES FOR SPREAD FOOTING DESIGN

Limit State	Nominal Bearing Resistance (psf)			
	Station 6+60	Station 8+00	Station 9+00	Station 10+00
Strength	8,700	11,900	17,200	31,400
Service (1/2 inch settlement)	1,100	1,300	1,600	2,100
Service (1 inch settlement)	2,200	2,700	3,200	4,100
Extreme	8,700	11,900	17,200	31,400

Notes:
1. Nominal bearing resistance should be multiplied by the appropriate resistance factors in Table 1, in order to determine the factored bearing resistance for design.
2. Nominal bearing resistances were determined using the design methodologies presented in the Washington Department of Transportation's Geotechnical Design Manual (GDM).

psf = pounds per square foot



0 1/4 1/2 1

Scale in Miles

King Street Station
4th Avenue Retaining Wall
Seattle, Washington

VICINITY MAP

June 2006 21-1-20406-002

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

NOTE

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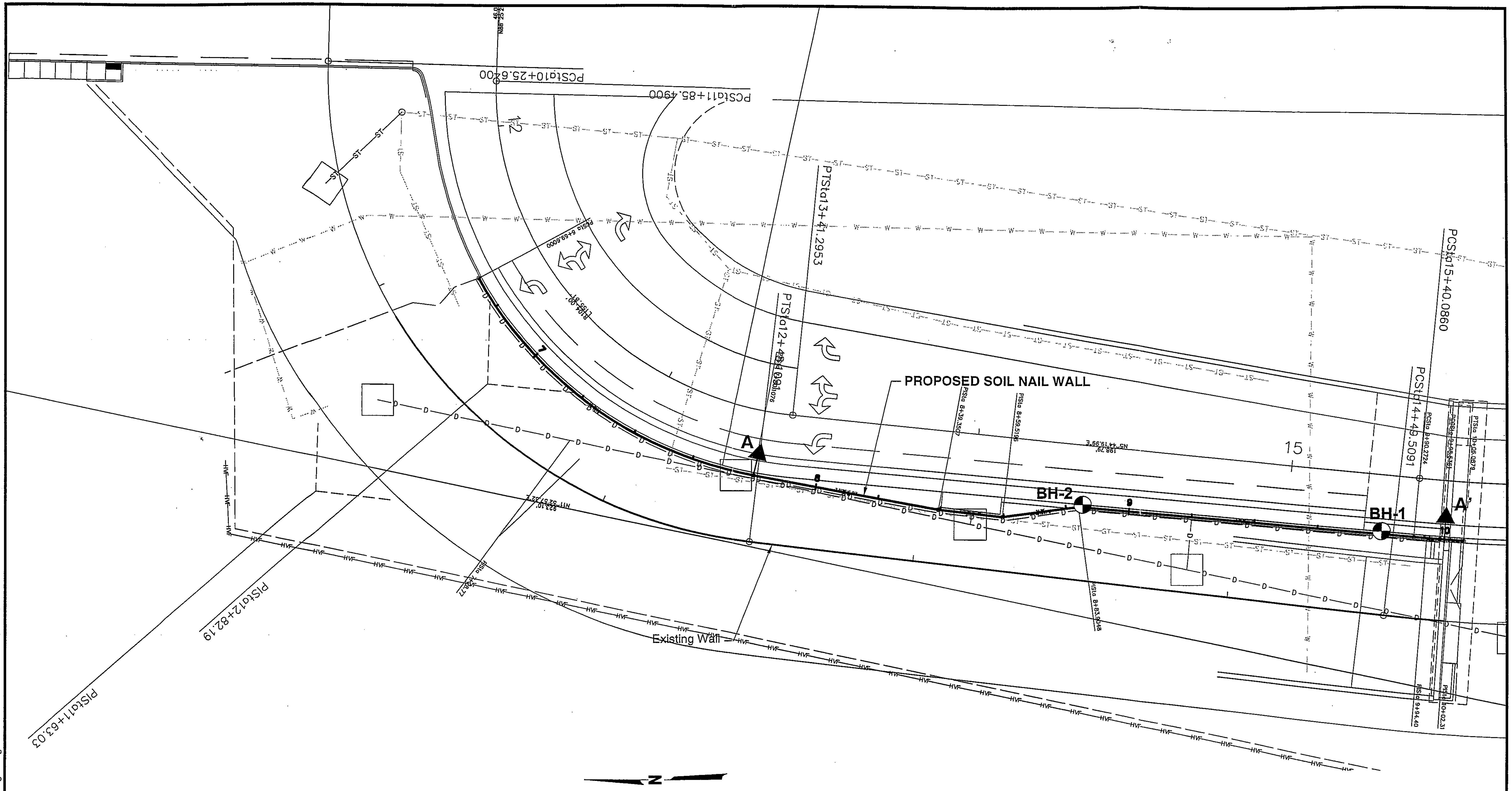
File: J:\211\20406-002\21-1-20406-002 Fig 1.dwg Date: 06-28-2006 Author: SAC



LEGEND

B-1 Boring Designation and Approximate Location

A Generalized Subsurface Profile Designation and Approximate Location

0 30 60

Scale in Feet

NOTES

1. Figure adapted from electronic file "KING_DESIGN_20051123.dwg" provided by HDR, Inc., received 6-26-06.
2. Location of existing wall is approximate.

King Street Station
4th Avenue Retaining Wall
Seattle, Washington

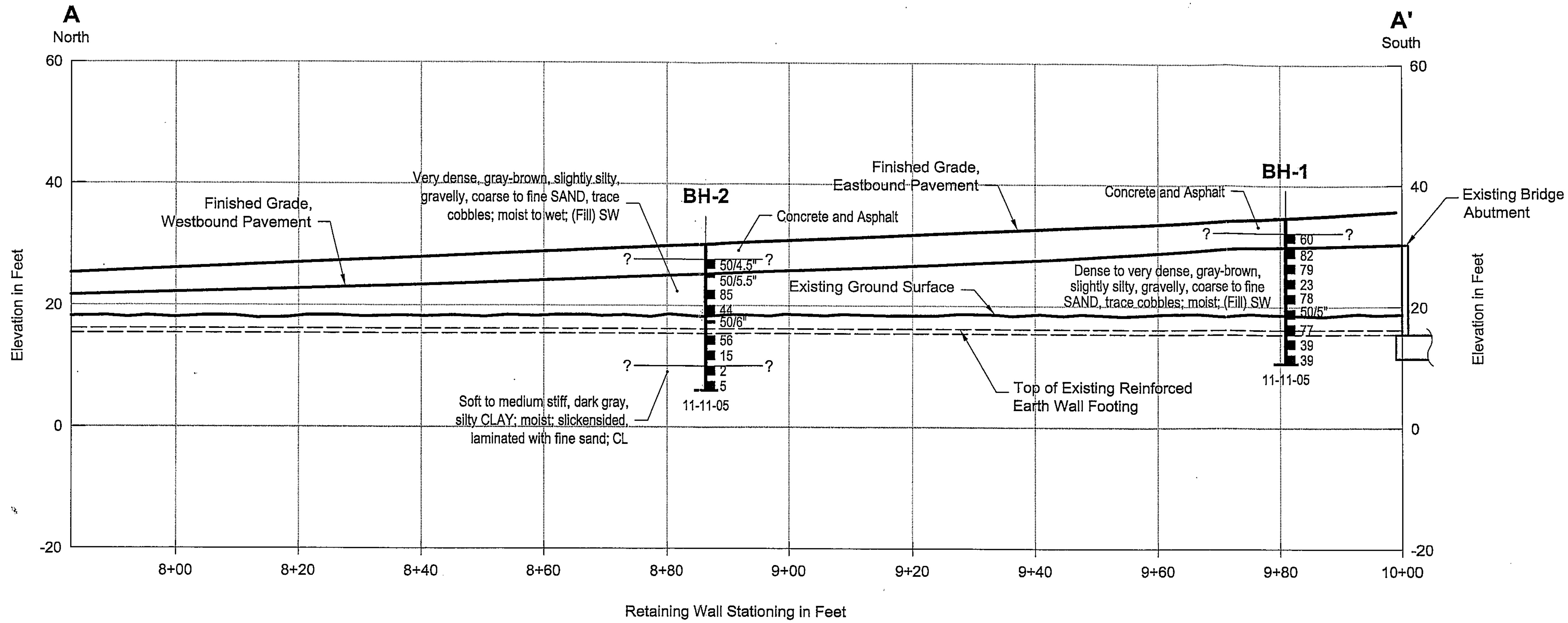
SITE AND EXPLORATION PLAN

June 2006 21-1-20406-002

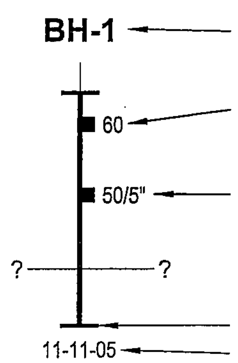
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2

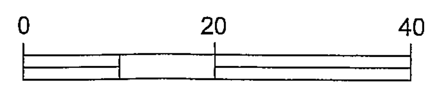
File: J:\211\20406-002\21-1-20406-002 Fig 2.dwg Date: 06-28-2006 Author: SAC



LEGEND



- Boring Designation
- Standard Penetration Test Blows/Foot
- Standard Penetration Test Blows/Inches Driven
- Approximate Geologic Contact
- Bottom of Boring
- Date Completed



Scale in Feet
Horizontal = Vertical

NOTES

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.
2. Boring locations are approximate.
3. Datum: NAVD 88

King Street Station
4th Avenue Retaining Wall
Seattle, Washington

GENERALIZED SUBSURFACE PROFILE A-A'

June 2006 | 21-1-20406-002

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FIG. 3

File: J:\211\20406-002\21-1-20406-002 Fig 3.dwg Date: 06-28-2006 Author: SAC

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WLI	Water level indicator

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND* - Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL* - Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	
Vibrating Wire	

King Street Station
4th Avenue Retaining Wall
Seattle, Washington

SOIL CLASSIFICATION AND LOG KEY

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FIG. 4
Sheet 1 of 2

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From ASTM D 2487-98 & 2488-93)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW	Well-graded gravels, gravels, gravel/sand mixtures, little or no fines.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines *(more than 12% fines)*	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sands *(less than 5% fines)*	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines *(more than 12% fines)*	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Inorganic silts of low to medium plasticity, rock flour, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic	OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH	Inorganic clays or medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.
2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND)* indicate that the soil may fall into one of two possible basic groups.

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SOIL CLASSIFICATION AND LOG KEY

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FIG. 4
Sheet 2 of 2

BORING_CLASS2 21-20406.GPJ SWNEW.GDT 6/28/06

Total Depth:	24 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	
Top Elevation:	~ 26 ft.	Easting:		Drilling Company:	Holt Drilling	Rod Type:	
Vert. Datum:		Station:		Drill Rig Equipment:		Hammer Type:	
Horiz. Datum:		Offset:		Other Comments:			

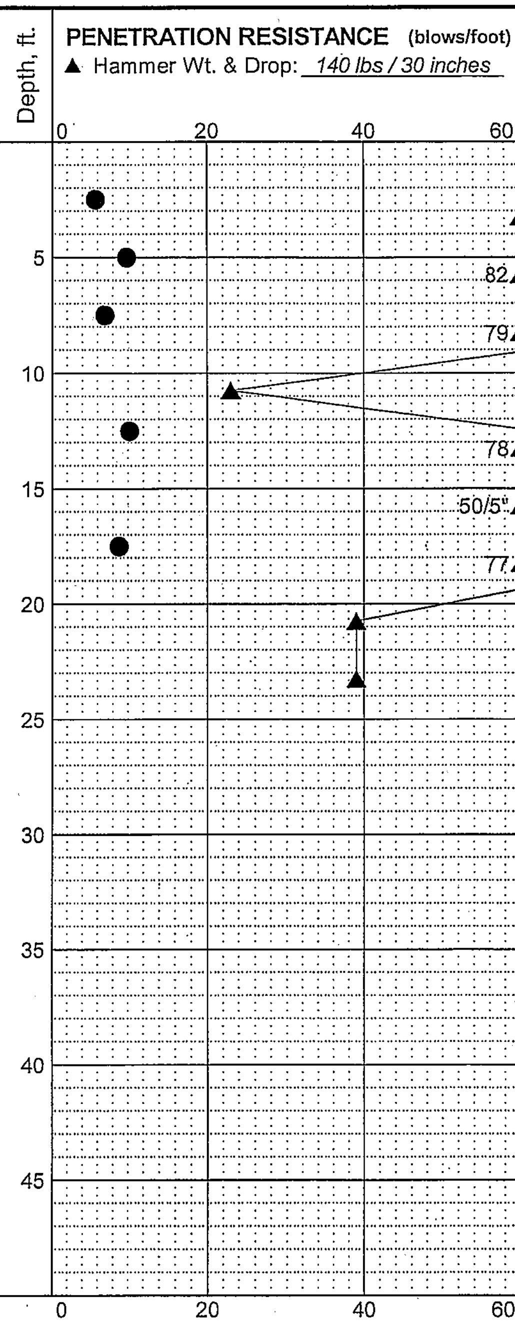
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Symbol	Samples	Ground Water
Concrete and asphalt.	2.5			
Dense to very dense, gray-brown, slightly silty, gravelly, coarse to fine SAND, trace of cobbles; moist; (Fill) SW.	24.0		1, 2, 3, 4, 5, 6, 7, 8, 29	None Observed During Drilling
BOTTOM OF BORING COMPLETED 11/11/2005				

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

* Sample Not Recovered

Standard Penetration Test

Plastic Limit ├──●──┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured using a cloth tape from existing site features and should be considered approximate.

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LOG OF BORING BH-1

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FIG. 5

MASTER LOG E 21-20406.GPJ SHAN WIL.GDT 6/28/06 Log: SLL Rev: SLL Typ: LKD

Total Depth:	*24 ft.*	Northing:		Drilling Method:	*Mud Rotary*	Hole Diam.:	
Top Elevation:	*~ 20 ft.*	Easting:		Drilling Company:	*Holt Drilling*	Rod Type:	
Vert. Datum:		Station:		Drill Rig Equipment:		Hammer Type:	
Horiz. Datum:		Offset:		Other Comments:			

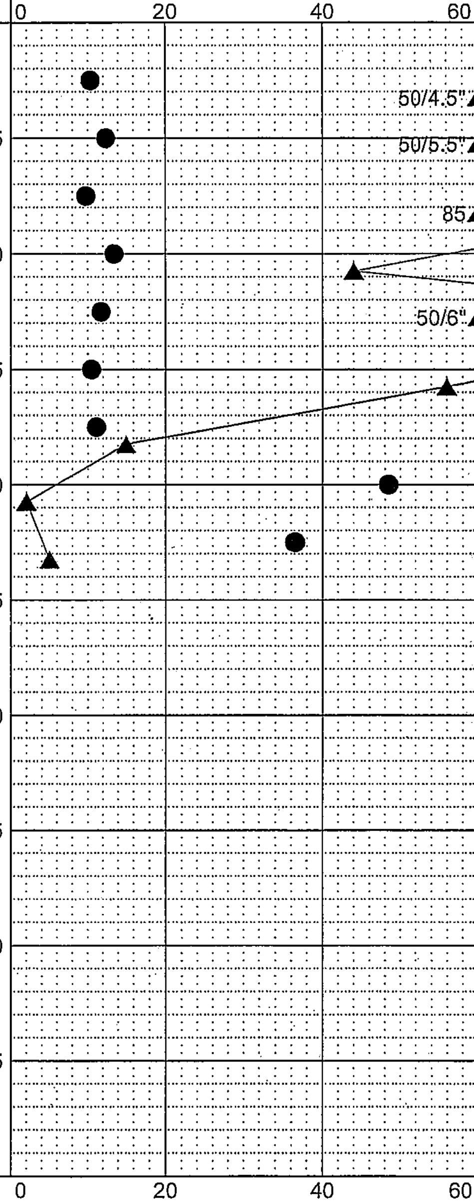
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.	Samples	Ground Water
Concrete and asphalt.			
Very dense, gray-brown, slightly silty, gravelly, coarse to fine SAND, trace of cobbles; moist to wet; (Fill) SW.	2.5	1, 2, 3, 4, 5, 6, 7	None Observed During Drilling
Soft to medium stiff, dark gray, silty CLAY; moist; slickensided, laminated with fine sand; CL.	20.0	8, 9	
BOTTOM OF BORING COMPLETED 11/11/2005	24.0		

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



LEGEND

* Sample Not Recovered

Ɪ Standard Penetration Test

Plastic Limit ├──●──┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured using a cloth tape from existing site features and should be considered approximate.

King Street Station
4th Avenue Retaining Wall
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LOG OF BORING BH-2

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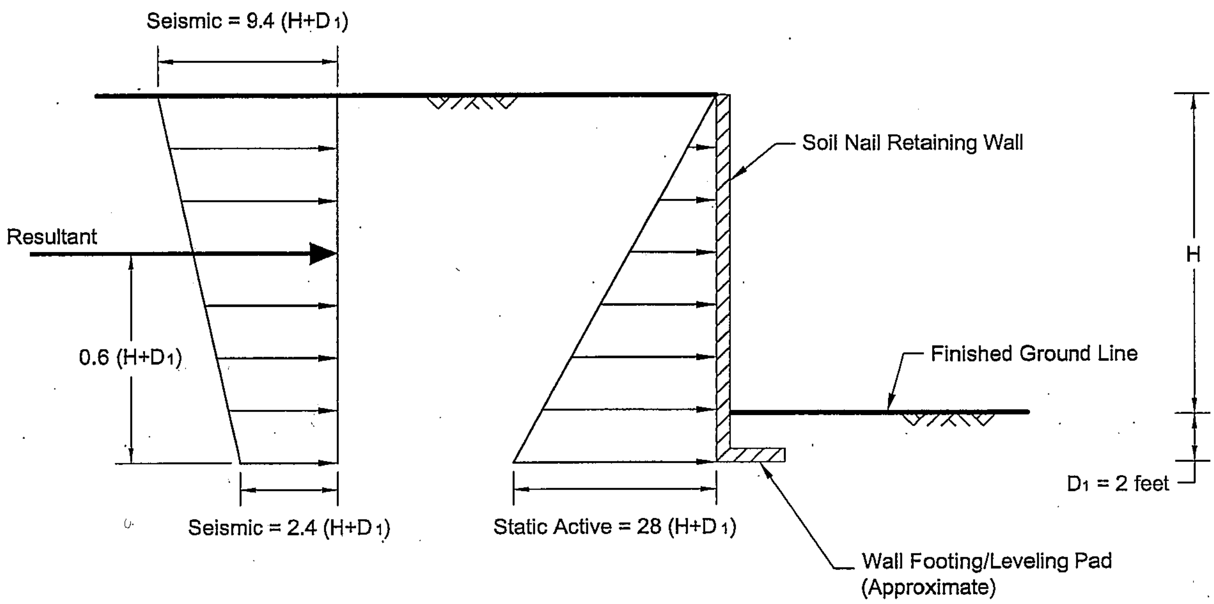
FIG. 6

MASTER LOG E 21-20406.GPJ SHAN WIL.GDT 6/28/06 Log: SLL Rev: SLL Typ: LKD

Recommended Earth Pressures
for Soil Nail Wall



Not to Scale

NOTES

1. All Earth Pressures are in units of Pounds per Square Foot.
2. Lateral pressures for traffic surface surcharges should be added to the earth pressures given above. See Figure 8.
3. The recommended pressure diagrams are based on a continuous wall system.
4. Free drainage assumed behind the wall.

LEGEND

H	=	Wall Height (Ft.)
D_1	=	Embedment Depths (Ft.)
$28 (H+D_1)$	=	Active Earth Pressure for Compacted Fill

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Seattle, Washington

SOIL NAIL WALL DESIGN CRITERIA
STA. 6+60 TO 10+06

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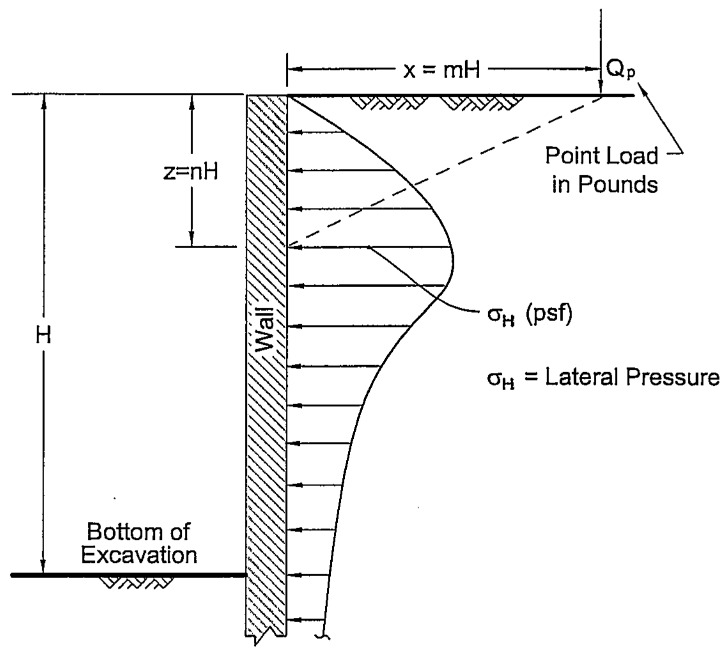
FIG. 7

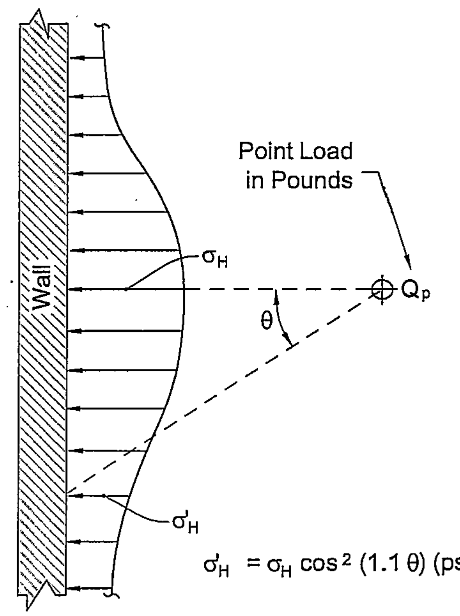
File: J:\211\20406-002\21-1-20406-002 Fig 7.dwg Date: 06-28-2006 Author: SAC



ELEVATION VIEW

For $m \le 0.4$: $\sigma_H = 0.28 \dfrac{Q_p}{H^2} \dfrac{n^2}{(0.16 + n^2)^3}$ (psf) (see Note 3)

For $m > 0.4$: $\sigma_H = 1.77 \dfrac{Q_p}{H^2} \dfrac{m^2 n^2}{(m^2 + n^2)^3}$ (psf)

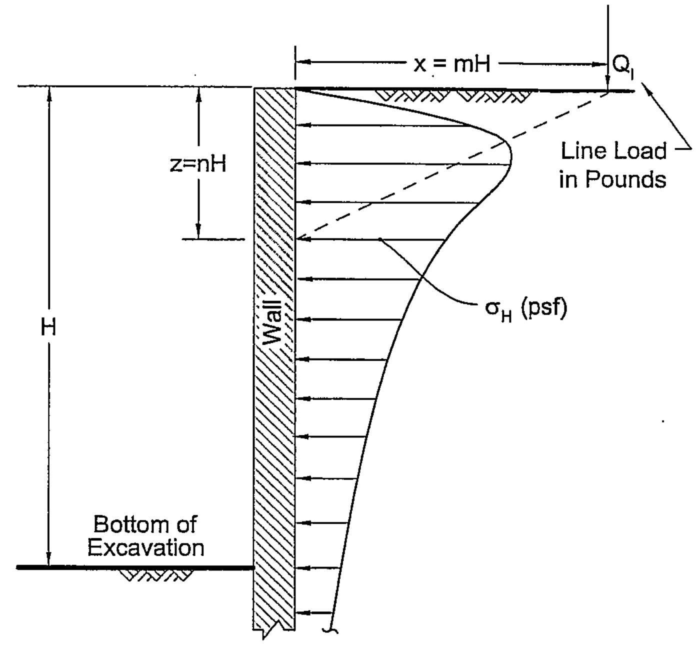


$\sigma'_H = \sigma_H \cos^2 (1.1\,\theta)$ (psf)

PLAN VIEW

A) LATERAL PRESSURE DUE TO POINT LOAD i.e. SMALL ISOLATED FOOTING OR WHEEL LOAD

(NAVFAC DM 7.2, 1986)



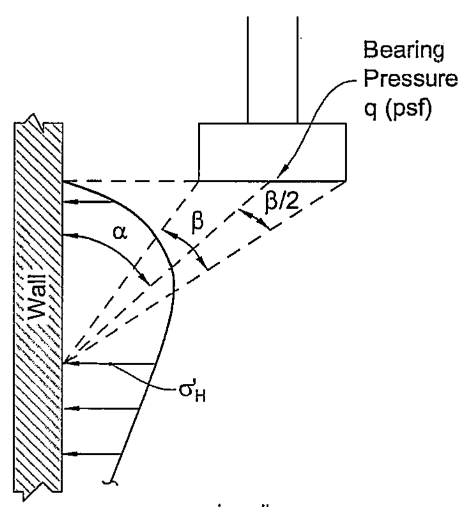
ELEVATION VIEW

For $m \le 0.4$: $\sigma_H = 0.20 \dfrac{Q_l}{H} \dfrac{n}{(0.16 + n^2)^2}$ (psf) (see Note 3)

For $m > 0.4$: $\sigma_H = 1.28 \dfrac{Q_l}{H} \dfrac{m^2 n}{(m^2 + n^2)^2}$ (psf)

B) LATERAL PRESSURE DUE TO LINE LOAD i.e. NARROW CONTINUOUS FOOTING PARALLEL TO WALL

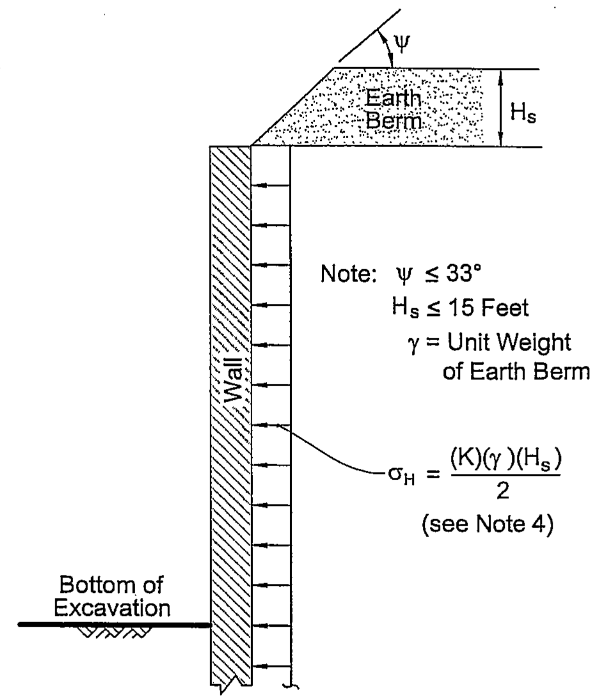
(NAVFAC DM 7.2, 1986)



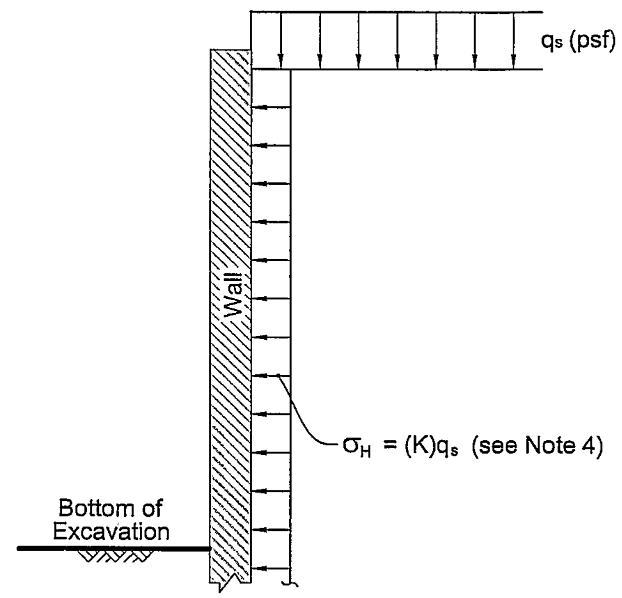
$\sigma_H = \dfrac{2q}{\pi} (\beta - \sin\beta \cos 2\alpha)$ (psf) (in radians)

C) LATERAL PRESSURE DUE TO STRIP LOAD

(derived from Fang, *Foundation Engineering Handbook*, 1991)



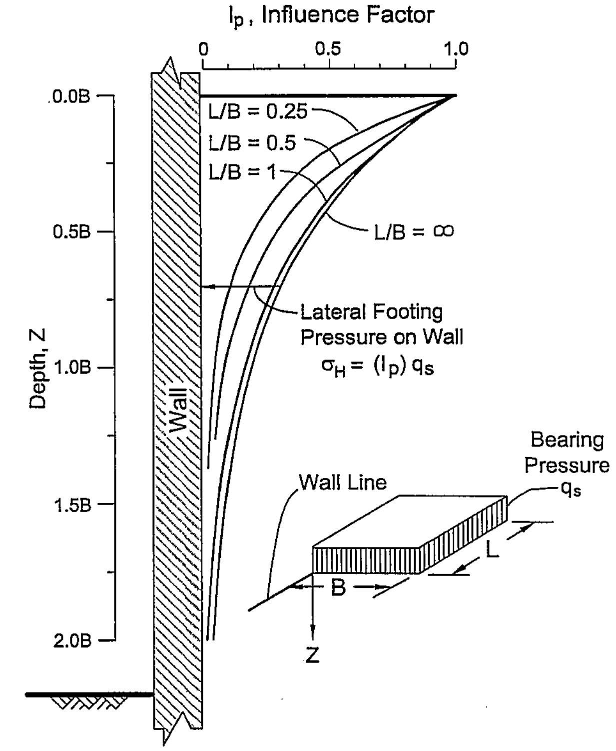
EARTH BERM



UNIFORM SURCHARGE

D) LATERAL PRESSURE DUE TO EARTH BERM OR UNIFORM SURCHARGE

(derived from Poulous and Davis, *Elastic Solutions for Soil and Rock Mechanics*, 1974; and Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, 1967)



E) LATERAL PRESSURE DUE TO ADJACENT FOOTING

(derived from NAVFAC DM 7.2, 1986; and Sandhu, *Earth Pressure on Walls Due to Surcharge*, 1974)

NOTES

1. Figures are not drawn to scale.
2. Applicable surcharge pressures should be added to appropriate permanent wall lateral earth and water pressure.
3. If point or line loads are close to the back of the wall such that $m \le 0.4$, it may be more appropriate to model the actual load distribution (i.e., Figure E) or use more rigorous analysis methods.
4. $K_a = 0.26$.

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4th Avenue Retaining Wall
Seattle, Washington

RECOMMENDED SURCHARGE LOADING FOR TEMPORARY AND PERMANENT WALLS

June 2006 | 21-1-20406-002

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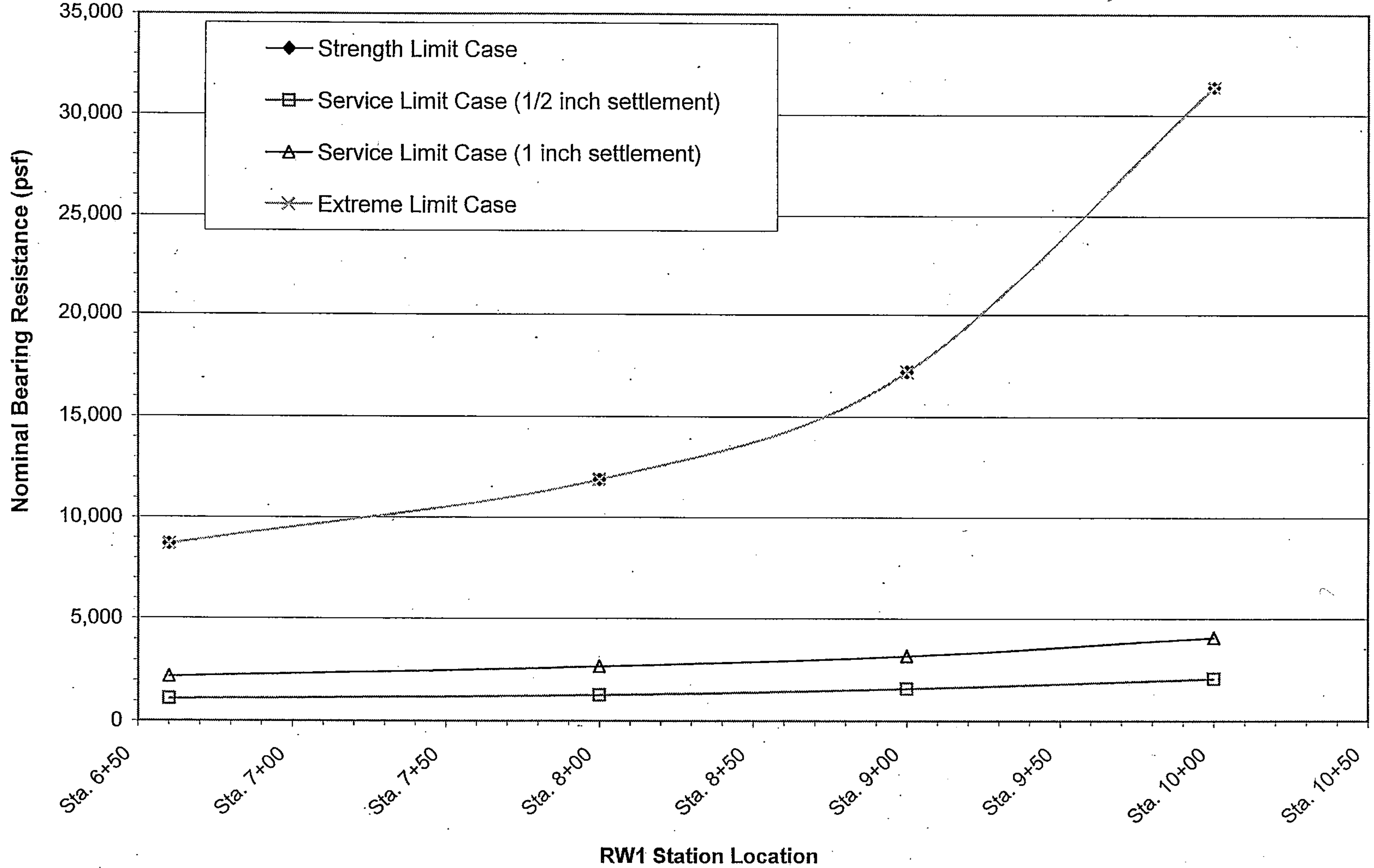
FIG. 8

File: J:\211\20406-002\21-1-20406-002 Fig 8.dwg Date: 06-28-2006 Author: SAC

Fig 9_21-1-20406-002.xls/lmm-6/28/2006



Notes:

1. Nominal bearing resistance should be multiplied by the appropriate resistance factors in Table 1, in order to determine the factored bearing resistance for design.
2. Nominal bearing resistances were determined using the design methodologies presented in the Washington Department of Transportation's Geotechnical Design Manual (GDM).

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4th Avenue South
Seattle, Washington

NOMINAL BEARING RESISTANCE OF MOMENT SLAB VERSUS STATION LOCATION

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FIG. 9

FIG. 9

Appendix

APPENDIX

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-20406-002

Date: June 28, 2006
To: HDR Engineering
Attn: Mr. Wayne Short

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland